1954

Welded interior beam-column connections, Proposal, 1954

C. D. Jensen

Lehigh University

INSTITUTE OF RESEARCH

233.7

Desk Copy

Proposal

Welded Interior Beam-Column Connections

824

by

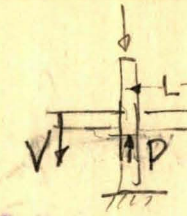
C.D. Jensen

Report No. 233.7

1. Fabricate in a shop
2. Proposition to Struc Stl. - def. rec.
3. Cost

⊗ COJ writes ltr (including cost) - send to Spraragen & Higgins at the same time (?)
⊗ TRA will approach AISC & then Struc Steel will get a package already financed.

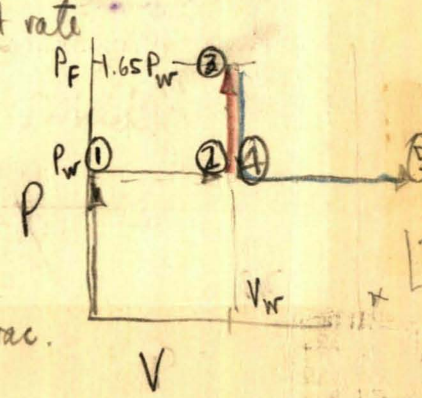
$\boxed{2/6/54}$ Thoughts after meeting



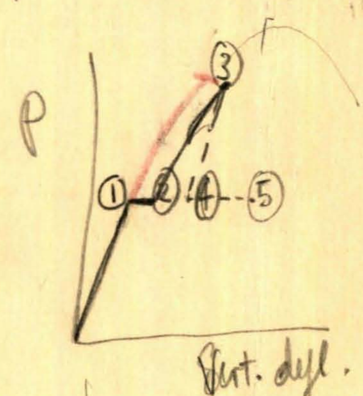
$\boxed{1}$

✓1. 25% overhead ... AISC project now at that rate
✓2. Proposed paper - he handled it right
3. change in loading sequence
   1. Emphasizes beam action
   2. Preserves two objectives
   3. maximum tensile deformation .... br. frac.
   4. Wind load would increase θ
   5. Plastic design
      looking for dependable moment strength
   6. Severe moment bending will do more damage to col than will severe P do to the com. ∴ more realistic to do P first

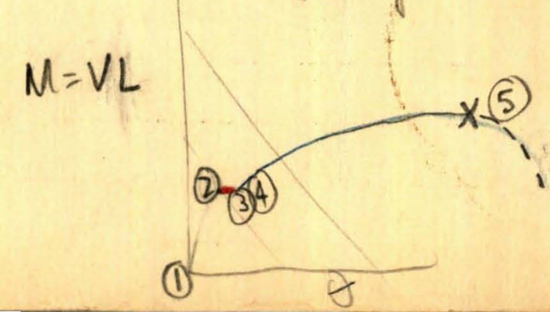
A. JT on design.



$\boxed{2}$



$\boxed{3}$



18 + 11 + 30 + 20
10"

LEHIGH UNIVERSITY
Department of Civil Engineering & Mechanics
Bethlehem, Penna.

File No. 233 9 February 1954

Mr. William Spraragen, Director
Welding Research Council
29 W. 39th Street
New York 18, New York

Subject: Subcommittee Report on Welded Interior Beam-Column Connections

Dear Sir:

Attached is the program for the investigation of the above connections as developed by the committee consisting of Messrs. L.S. Beedle, F.H. Dill, T.R. Higgins and Carl Kreidler, with myself as chairman.

At the meeting of the Committee February 5, 1954 at Lehigh University at which the final details were agreed upon, the Committee gave enthusiastic endorsement to this program. The Committee is appreciative of the contributions made by Jonathan Jones who attended the above meeting by invitation.

Very truly yours,

Cyril D. Jensen, Chairman
Professor of C.E.

CDJ:bk
Enc:
cc: Members of Committee
Messrs: Higgins
Dill
Kreidler
Lawson
Beedle

233

Jones

* loss of ductility of connection

LEHIGH UNIVERSITY
The Institute of Research
Fritz Engineering Laboratory
Bethlehem, Pennsylvania

WELDED INTERIOR BEAM-COLUMN CONNECTIONS

Introduction:

A research project is proposed at Lehigh University to study restraining beam-column connections of both the direct-welded type and the type in which the beam is mounted on a seat angle or bracket and the top of the beam is secured to the column by means of a top plate. This proposal is an extension of a number of research projects conducted at Lehigh University by Bruce Johnston and associates and of a project by Brandes and Mains, Report of Tests of Welded Top-Plate and Seat Building Connections (A.W.S. Journal, March 1944) and in the studies by Yang, Beedle, and Johnston on welded connections in the A.W.S. Journal for April 1952.

The above researches on restraining beam-column connections have not been carried to the point where definite conclusions suitable for the designer could be reached. In particular, information is lacking on the effect of restraining connections on column load capacity and also as to whether or not column stiffening is required and, when needed, how to design it. Information is also lacking concerning the designer's ability to estimate the moment-rotation curve, the degree of restraint, and the reserve strength of a designed assembly. Phase I of this proposed program is designed to obtain information on these items. Heretofore tests of beam-column connections have usually disregarded the axial column loads; in the present tests it is proposed that the column be additionally subjected to an axial compression comparable to that existing in practice.

In the top plate and seat type of restraining beam-column connection there is the additional need to know the design requirements that will provide ductile behavior of the top plate. An examination is especially needed of the full implications of the statement in the AISC Standard Specifications for Steel Buildings "except that some non-elastic but self-limiting deformation of a part of the connection may be permitted when this is essential to the avoidance of overstressing of a weld". This should be made on top plate connections at various working stresses from 20,000 psi. to the yield point.

The value of restraining beam-column connections in design has been indicated in the work of Brandes and Mains who showed a good possibility that designers could use WL/12 rather than WL/8 for design of beams (effecting a savings in beam weight) provided that the connections will develop at least 75% of full end restraint.

Examination is needed of other factors such as the effect of wind moments and the behavior of a four-way connection at a column. The effect of beams framing to a column from two or three sides such as takes place at a corner or side column was partially treated in the Brandes-Mains paper, but not in such sufficient detail as to be useful to the designer.

Development of information concerning top plates, four-way connections, effects of wind moments, and application to design is somewhat dependent on the findings of Phase I of the proposed program. The proposal for the investigation of these items is consequently set forth as possible future work in Phases II to V of this proposal.

PROPOSAL

It is proposed to make an evaluation of restraining beam-column connections of both the direct-welded type and the top plate and seated type, these two being economically competitive in commonly used sizes of beams. The work is to be performed in the steps outlined below and is to be under the guidance of an advisory group of engineers. Attention is limited in Phase I primarily to the study of what is considered to be the most important practical problem: Column stiffening requirements. In Phase II attention is directed primarily to the behavior of the four-way connection.

IMMEDIATE PROGRAM

I. Two-Way Connection - Direct-Welded

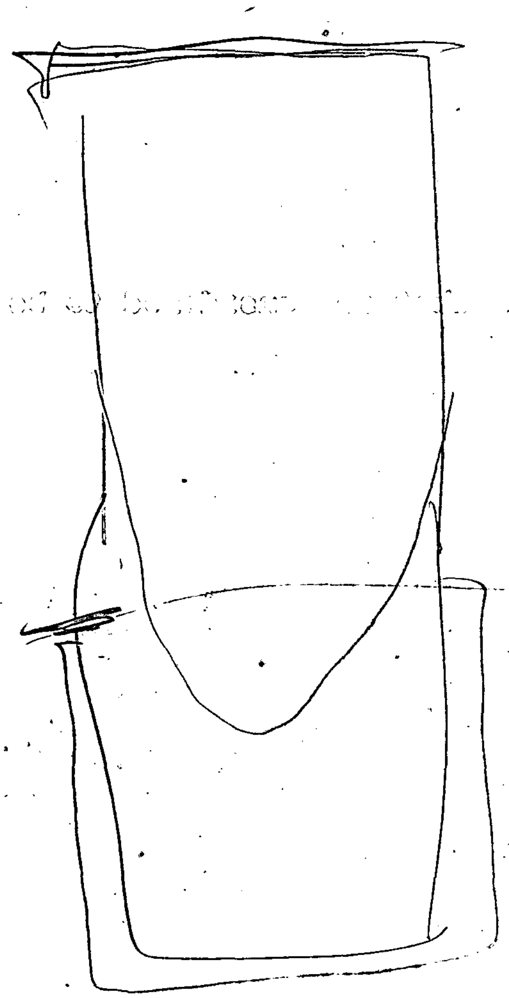
Design, preparation, and testing of specimens similar to Fig. 1 for the purpose of determining the behavior and stress distribution in the columns. The following series of tests are proposed for this phase. (They are summarized in Table 1): In particular it is desired to know what percentages of the beam flange area should be provided as stiffening on the column web to determine how much flange load is carried by the column web.

A. Series A. All beams to be 16 WF 36 and to be direct-welded to columns as shown in Fig. 1 (or as modified <u>except</u> that the column stiffeners are to be <u>omitted</u>.

1) Specimen 1. Column to be 8 WF 31
2) Specimen 2. Column to be 8 WF 67
3) Specimen 3. Column to be 12 WF 40

B. Series B. Same as Series A except for the inclusion of the column stiffeners. It is presently planned to make the stiffeners the same thickness as the beam flange.

C. Series C. Same beams and columns as in Series A, but place the column stiffeners parallel to the column web as in Fig. 1-B.



Sequence as of 4/29/55

A = 10.59

TABLE 1: PROGRAM OF RESTRAINED BEAM TESTS - TWO-WAY DIRECT-WELDED CONNECTIONS

Series	Test No.	Column Size	Web "w"	Flg. "t"	Beam Size	Web "w"	Flg. "t"	Stiffening Type	Stiffening Dimensions	Joint Design
A	A 1 3/12/55	8WF31	.288	.433	16WF36	.299	.428	None	- - -	F.1-A
	2	8WF67	.575	.933	"	"	"	"	- - -	"
	3	12WF40	.294	.516	"	"	"	"	- - -	"
	4	12WF65	.390	.606						
	5*	12WF99	.580	.921						
B	B 6 4/6/55	8WF31	.288	.433	"	"	"	Fig. 1	3.5x7/16	F.1-A
	7**	8WF67	.575	.933	"	"	"	"	"	"
	8	12WF40	.294	.516	"	"	"	"	"	"
C	C 9 5/7/55	8WF31	.288	.433	"	"	"	Fig.1-B	7.1x5/16x22	F.1-B
	10**	8WF67	.575	.933	"	"	"	"	7.2x1/2x22	"
	11	12WF40	.294	.516	"	"	"	"	10.9x5/16x22	"

* Column weight may be changed dependent on results of previous tests
** May be omitted dependent on results of test of A-2

.25"

POSSIBLE FUTURE PROGRAM

II. Four-Way Connection

Design, preparation, and testing of four-way beam-column clusters similar to Fig. 1 except for the addition of two beams framing into the web of the column. The designs would include both direct-welded and the top-plate and seated type of connections. The purpose of these tests is to study the behavior of the column and connections under the four-way loading.

2/21/59
1) Report on 3
2) List the problem
3) Analyt tools
4) Cond tests
5) Test Progr.

III. Top-Plate Connections

Following a review of previous top plate tests, there will be preparation and testing of any new or improved designs of top plates. (It is presently believed that this can be accomplished by simple tension tests (Fig. 2) with perhaps a few confirming two-way or four-way beam-column tests as in Fig. 1).

IV. Effects of Wind Moments

A study of the literature and of the tests being conducted presently at Lehigh University, may provide sufficient material to indicate proper design methods for including the effect of wind moments in combination with gravity loadings.

V. Suggested Standard Designs

Preparation of suggested standard designs, including an estimate of the moment-rotation curve for each design, the plastic or reserve strength, and other limiting conditions. Connections having top plates will be examined in designs at various working stresses from 20,000 psi to the yield point with beams and welds at allowable (AISC) stresses.

Suggested Testing Procedure

Gages:

1. SR-4 gages on centerline of column web spaced at about 4", also A-1 type SR-4 gages on stiffeners and on top flange of one beam near column.

2. Rotation bars on beams and on column web close to the column flanges.

3. Whitewash

Loading:

1. Apply increment of loading to the tension jacks. Adjust column load from head of testing machine to provide a stated working stress in the lower portion of the column. This column stress will be computed from the A.I.S.C. column formula.

2. Read gages after creep strains are essentially removed.

3. Repeat increments of loading until the moment rotation curve crosses the 2X beam line for the assumed uniformly loaded beam span that this cantilever test is simulating.

4. Reduce the loads on the cantilever beams until they are equal to the end reactions of the simulated uniformly-loaded beam at working load. Then add increments of axial load to the column until failure of the column is observed.

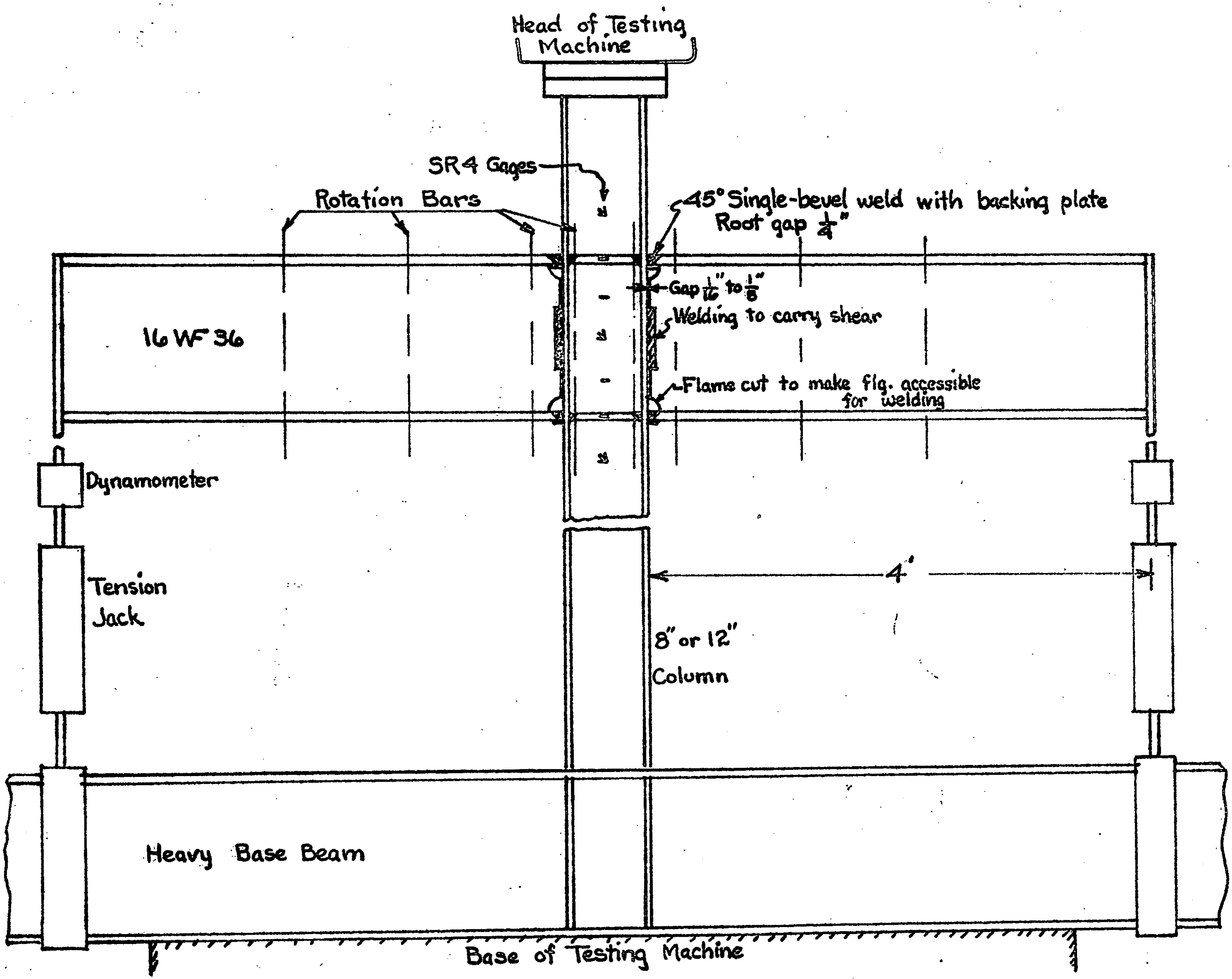


FIG. 1 PROPOSED TEST ARRANGEMENT TWO-WAY BEAM-COLUMN CONNECTION

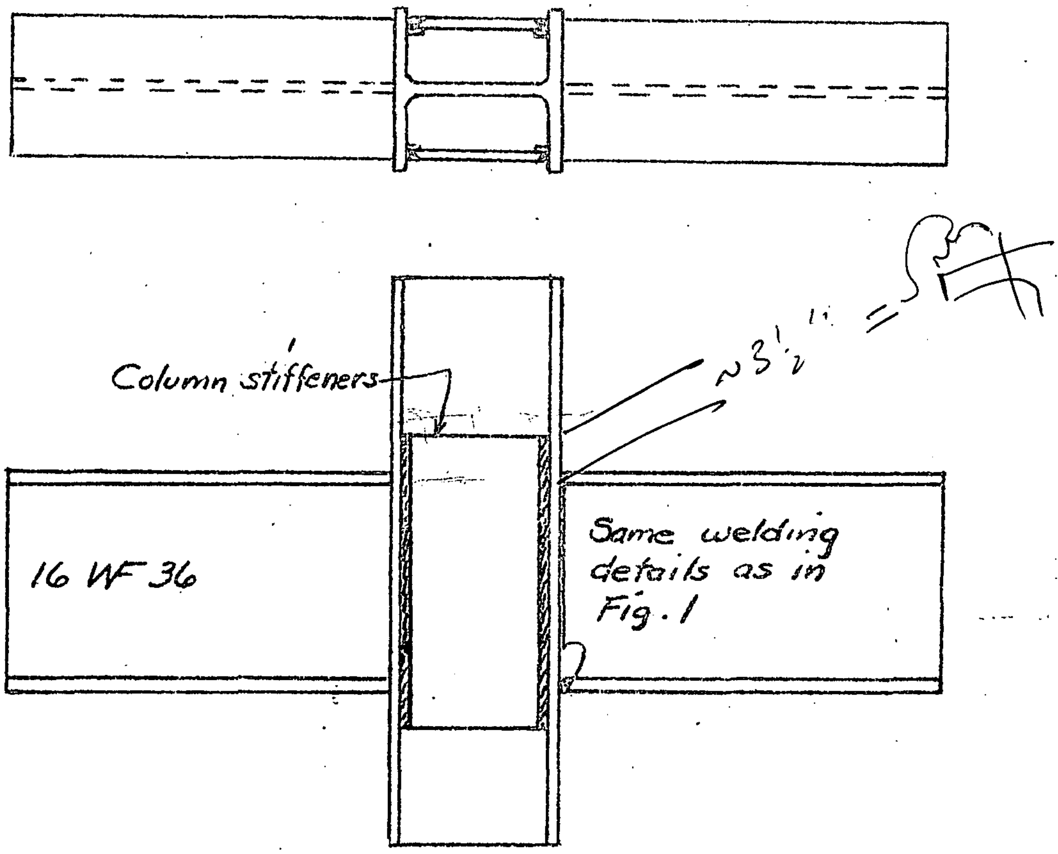


FIG. 1- B. COLUMN STIFFENERS PARALLEL TO WEB OF COLUMN

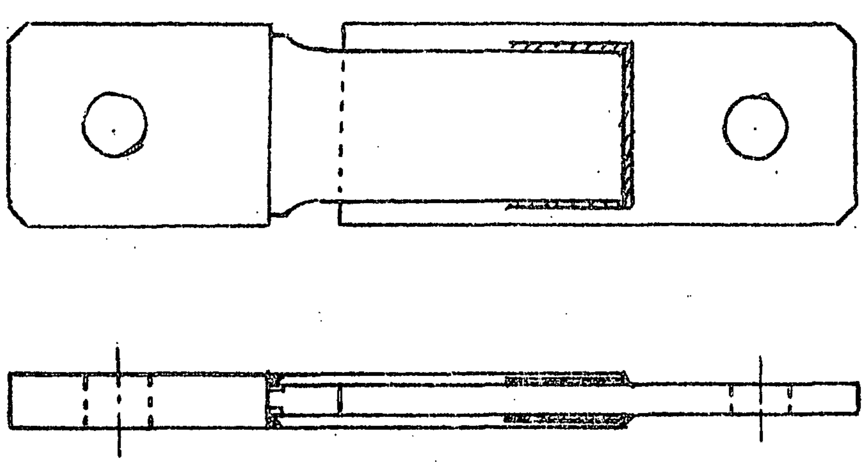


FIG. 2. TOP PLATE TENSIONS SPECIMENS

Estimated Budget for First Year of Program
(1 September 1954 to 30 June 1955)

A. Salaries and Wages

1. Supervision and services, part-time, Prof. C. D. Jensen and Assoc. Prof. L. S. Beedle	$1,120.00
2. Graduate research assistant, half-time	1,600.00
3. Machinists and other labor, secretarial and clerical help, hourly wages	1,480.00
Total salaries and wages	$4,200.00
B. Overhead, undistributed costs for the use of research facilities, 33-1/3 per cent of salaries and wages. The actual overhead determined by the Army Audit Agency for Lehigh University for the period 1 Nov. 1953 to 31 Oct. 1954 is 61 per cent. Part of the contribution of Lehigh University to this program is in the acceptance of a lower overhead rate.	$1,400.00
C. Consumable materials, supplies, etc.	$1,000.00
Total	$6,600.00

It is not anticipated that the research program outlined in this proposal can be completed in one year. If the results at the end of that period offer sufficient promise a request for a continuation of the project will be submitted.

LEHIGH UNIVERSITY
Bethlehem, Penna.

File No. 233

Fritz Laboratory
11 February 1954

M E M O R A N D U M

Interior Beam-Column Connections
ALTERNATE LOADING PROCEDURE

This note is to suggest a change in the loading sequence for the tests on restraining beam-column connections. It will allow us to obtain substantially the same information as before and, in addition, further information about the behavior of the connections.

The suggested modification is as follows: After the working load is applied to the column and then to the beam, a "full load" equal to 1.65 times the working load would be applied to the column. This load would then be reduced to the working load and, keeping column load constant, the connection would then be loaded to collapse. The difference between this loading procedure and the one that was selected on Friday, February 5, is that the <u>connection</u> would be loaded to collapse with working load on the column instead of collapsing the <u>column</u> with working load on the connection. We would <u>still</u> be able to observe the behavior of the joint with respect to two important conditions:

(1) Would the connection loaded to working load prevent the column from carrying the full load (working load times factor of safety)

(2) Can the connection develop adequate reserve strength while the column is loaded with normal working load.

I believe these were the two facets of the problem that the committee considered important.

The following seem to me to be the most important advantages of the procedure I am suggesting:

(1) It emphasizes connection action, still preserving the aspects of column influence.

(2) The connection would be loaded to complete failure. Hence we could observe any possible weld failures or plate fractures ... and it is important to know how "ductile" our connections are.

(3) Overloading the connection first will cause more severe deformation in the joint than will overloading the column first. Such severe overloading seems unrealistic.

In summary, then, the suggested alternate loading procedure would be as follows (refer to figures attached for the points that correspond to steps below.).

1. Apply working load (P_W) to the column. (Fig. 2) Note that P is the load in the column below the connection.

2. Apply working load (V_W) on the connection. Point 2, Fig. 2. V_W is determined from the M - θ curve of Fig. 4.

3. Apply full load to column. $P_f = 1.65 \times P_W$. Keep working load on connection.

4. Unload column to working load, P_W.

5. Load connection to collapse (Fig. 4).

Lynn S. Beedle

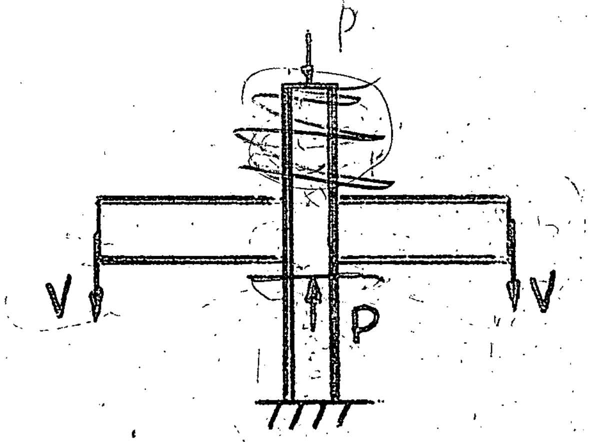


FIG 1

Joint Assembly

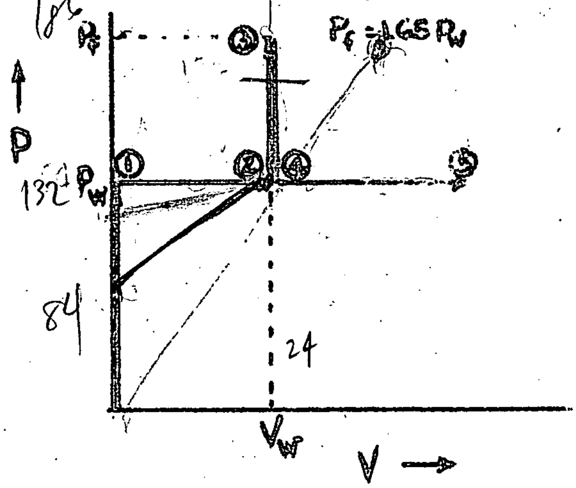


FIG 2

Relationship of Column Load to Connection Load

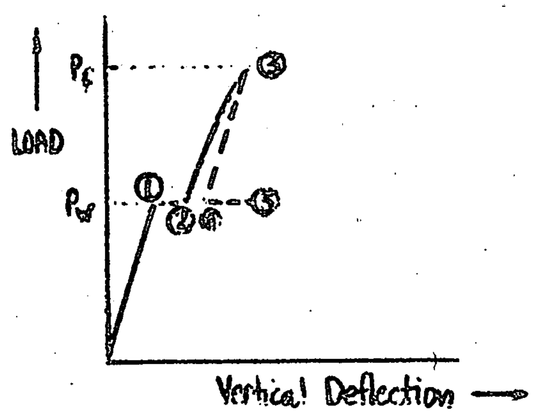


FIG 3

Load vs Vertical Deflection of Column

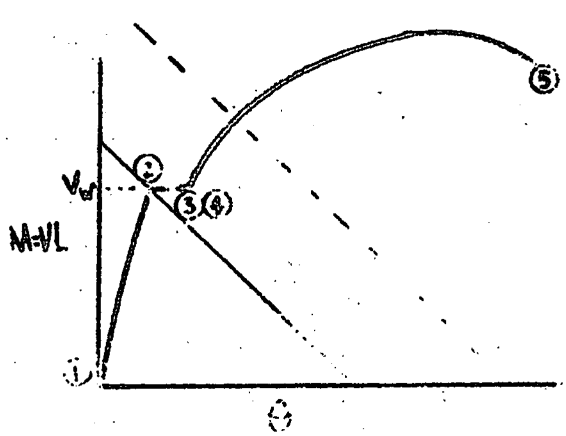


FIG. 4

Moment vs Rotation curve of the Connection.

File 233.7

LEHIGH UNIVERSITY
Department of Civil Engineering & Mechanics
Bethlehem, Pennsylvania

December 12, 1953

Mr. F. H. Dill, Welding Engineer
Mechanical Engineering Department
American Bridge Company
Ambridge, Pennsylvania

PROPOSAL

Mr. T. R. Higgins, Director of Engineering
American Institute of Steel Construction
101 Park Avenue, New York, New York

Mr. Carl L. Kreidler, Chief Structural Engr.
Lehigh Structural Steel Company
Allentown, Pennsylvania

Mr. Heath Lawson, Consulting Engineer
1677 Rensselaer Road
West Englewood, New Jersey

Mr. Lynn S. Beedle, ~~Assistant Engineer~~ Director ← THIS COPY FOR
Fritz Engineering Laboratory
Lehigh University
Bethlehem, Pennsylvania

Gentlemen:

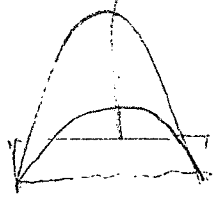
Herewith is a revised draft of our proposed WELDED INTERIOR BEAM-COLUMN CONNECTIONS project. In this rewrite I have endeavored to incorporate the various editorial changes and a few "connection" detail changes (the latter suggested by Mr. Kreidler). In addition, I have attempted to incorporate Mr. Higgins' request that the proposal be more detailed and that the column be loaded to simulate practice.

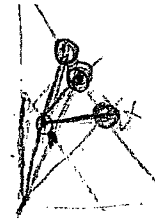
The question now arises as to whether a meeting of the committee is needed or whether we can handle the remaining details by mail.

Very truly yours

C. D. Jensen
Chairman

CDJ:EY





TRH suggests para on
Importance of stiffening

File 233.7

LEHIGH UNIVERSITY
Department of Civil Engineering & Mechanics
Bethlehem, Penna.
December - 1953

~~PROPOSAL~~ - WELDED INTERIOR BEAM-COLUMN CONNECTIONS

A fellowship is proposed at Lehigh University to study restraining beam-column connections of both the direct-welded type and the type in which the beam is mounted on a seat angle or bracket and the top of the beam is secured to the column by means of a top plate. This proposal is an extension of a number of research projects conducted at Lehigh University by Bruce Johnston and associates and in a project by Brandes and Mains, Report of Tests of Welded Top-Plate and Seat Building Connections (A.W.S. Journal, March 1944) and in the studies by Yang, Beedle and Johnston on welded connections in the A.W.S. Journal for April 1952.

The above researches on top plate and seat connections have not been carried to the point where definite conclusions suitable for the designer could be reached, information being lacking on criteria as to whether or not column stiffening is required, and when needed, how to design it. Information is also lacking concerning the designer's ability to estimate the reserve strength or the factor of safety of a designed assembly. Criteria are also lacking on the design procedures to assure ductile behavior of the top plates. The work of Brandes and Mains showed a good possibility that designers could use WL/12 rather than WL/8 for design of beams provided the connection could carry WL/16. All that is needed, for example, is further proof of the rather tentative conclusion of the above writers that the moment-rotation curve can be predicted with reasonable accuracy, that it have a reasonable moment strength, and a stiffness greater than 75% of that obtaining for a fully fixed connection.

In regard to the direct-welded beam-column connection the importance of knowing when column stiffening is required is perhaps of even greater importance than for the top-plate connection. Further, researches at Lehigh University ~~by Yang~~ indicate that for the complete restraint which obtains in the case of a direct-welded connection, the reserve strength of the assembly may be reduced somewhat through local buckling of the beam flanges immediately adjacent to the connection. In summary, there appears to be a real need to determine the factors involved in obtaining a good beam-column connection design and to establish a design procedure. Heretofore tests of beam-column connections have usually disregarded the axial column loads; in the present tests it is proposed that the column be additionally subjected to an axial compression comparable to that existing normally in practice.

Examination is needed of other factors such as the effect of wind moments and the behavior of a four-way connection at a column. The effect of beams framing to column from two or three sides such as takes place at a corner or side column was partially treated in the Brandes-Mains paper, but not in sufficient detail as to be useful to the designer.

An examination is especially needed of the full implications of the provision in the AISC, Standard Specifications for Steel Buildings "except that some non-elastic but self-limiting deformation of a part of the connection may be permitted when this is essential to the avoidance of overstressing a weld". This should be made

Mr Higgins

Two plans - conven. design
new design moment -- restrained

all agree these schools of thought exist

Mr. Jones introduced idea of a control column

Possibility of a control column

→ Proportional loading is needed and will be used

Tier building example variation from top to bottom of bldg

Is there anything wrong with > 75%. Plastic excuse for an elastic design on top-plate connections at various working stresses from 20,000 psi. to the yield point.

The reserve strength of the connections would be a definite part of the investigation. Through knowledge obtained in the researches into the plastic strength of beams at Lehigh University and elsewhere, it is believed that a fair estimate can be made of the moment-rotation characteristics and of the reserve strength of each type of connection.

PROPOSAL

It is proposed to make an evaluation of restraining beam-column connections of both the direct-welded type and the top plate and seated type, these two being economically competitive in commonly used sizes of beams. The work is to be performed in the steps outlined below and is to be under the guidance of an advisory group of engineers. Attention is limited primarily in Phase I to a study of what is considered to be the most important practical problem: Column stiffening requirements. In Phase II attention is directed primarily to the behavior of the four-way connection.

IMMEDIATE PROGRAM

I **Two-Way Connection - Direct-Welded**

Design, preparation, and testing of specimens similar to Fig. 1 for the purpose of determining the behavior and stress distribution in the columns. The following series of tests are proposed for this phase. (They are sumarized in Table 1): In particular it is desired to know what percentage of the beam flange area should be provided as stiffening on the column web to determine how much flange load is carried by the column web.

A. Series A. All beams to be 16 WF 36 and to be direct-welded to columns as shown in Fig. 1 (or as modified) except that the column stiffeners are to be omitted.
   1) Specimen 1. Column to be 8 WF 31
   2) Specimen 2. Column to be 8 WF 67
   3) Specimen 3. Column to be 12 WF 40

B. Series B. Same as Series A except for the inclusion of the column stiffeners. It is presently planned to make the stiffeners the same thickness as the beam flange.

C. Series C. Same beams and columns as in Series A, but place the column stiffeners parallel to the column web as in Fig. 1-B.

#A3 is parallel with t & w of 8WF31

#B5 and C8 might not be needed

Do #1-4-7 in series
Save some 8WF31 for phase $\boxed{B}$ to see if top plate acts more flexibly to protect fraction of weld

TABLE 1: PROGRAM OF RESTRAINED BEAM TESTS - TWO-WAY DIRECT-WELDED CONNECTIONS

Series	Test No.	Column Size	Web "w"	Flg. "t"	Beam Size	Web "w"	Flg. "t"	Stiffening Type	Stiffening Dimensions	Joint Design
A	1	8WF31	.288	.433	16WF36	.299	.428	None	- - -	F.1-A
	2	8WF67	.575	.933	"	"	"		- - -	"
	3	12WF40	.294	.516	"	"	"	"	- - -	"
	3a	12WF65								
	3b	12WF ×								
B	4	8WF31	.288	.433	"	"	"	Fig.1	3.5x7/16	F.1-A
	5	8WF67	.575	.933	"	"	"	"	"	"
	6	12WF40	.294	.516	"	"	"	"	"	"
C	7	8WF31	.288	.433	"	"	"	Fig.1-B	7.1x5/16x22	F.1-B
	8	8WF67	.575	.933	"	"	"	"	7.2x1/2x22	"
	9	12WF40	.294	.516	"	"	"	"	10.9x5/16x22	"

$\boxed{\text{POSSIBLE FUTURE PROGRAM}}$

II Four-Way Connection

Design, preparation, and testing of four-way beam-column clusters similar to Fig. 1 except for the addition of two beams framing into the web of the column. The designs would include both direct-welded and the top-plate and seated type of connections. The purpose of these tests is to study the behavior of the column and connections under the four-way loading.

III Top-Plate Connections

Following a review of previous top plate tests, there will be preparation and testing of any new or improved designs of top plates. (It is presently believed that this can be accomplished by simple tension tests (Fig. 2) with perhaps a few confirming two-way or four-way beam-column tests as in Fig. 1).

IV Effects of Wind Moments

A study of the literature and of the tests being conducted presently at Lehigh University, may provide sufficient material to indicate proper design methods for including the effect of wind moments in combination with gravity loadings.

V Suggested Standard Designs

Preparation of suggested standard designs, including an estimate of the moment-rotation curve for each design, the plastic or reserve strength, and other limiting conditions. Connections having top plates will be examined in designs at various working stresses from 20,000 psi to the yield point with beams and welds at allowable (AISC) stresses.

⊗ 12WF99 matches t & w of 8WF67. It would depend, however, on the results of the 8WF67

OUTLINE ON PROJECT

Phase	Subject	Applicable Tests	Variables	Constants
I	Two-Way connections - Direct Welded A. Design of Stiffeners in Column 1. Criteria for design - Loads 2. " " stiffener details (thickness & arrangement) B. Strength of Connection 1. Yield strength 2. Ultimate strenght C. Stiffness of Connection Moment-rotation characteristics	(See Table 1) 1-9 inclusive 1-9 inclusive 1-9 inclusive	1 Column Section 2 Stiffening details	Column load Fix 1 Beam Section 2 Joint Design
II	Four-Way Connection (Beam-to-Col. Flg. Connections Direct Welded, Beam-to-Col. Web Connections Modified as necessary) A. Design of Stiffeners in Column 1. Criteria for design - Loads 2. " " stiffener details (thickness & arrangement) B. Strength of Connection 1. Yield strength 2. Ultimate strength C. Stiffness of Connection Moment-rotation characteristics	To be proposed To be proposed To be proposed	1 Column Size 2 Stiffening details 3 Joint design	Column load Beam size
III	Top Plate Connection	To be proposed		
IV	Effects of Wind Moments	Study of related tests		
V	Suggested Standard Designs	None (unless a few confirming tests are indicated)		

Suggested Testing Procedure

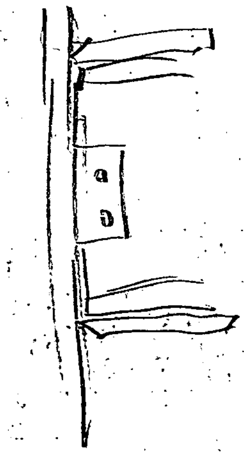
Gages:

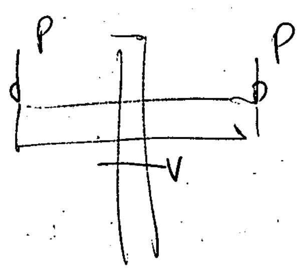
1. Rosette-type SR4 gages on centerline of column web spaced at about 4", also A-1 type SR4 gages on stiffeners.
2. Rotation bars on beams and on column web close to the column flanges.
3. Whitewash

Loading:

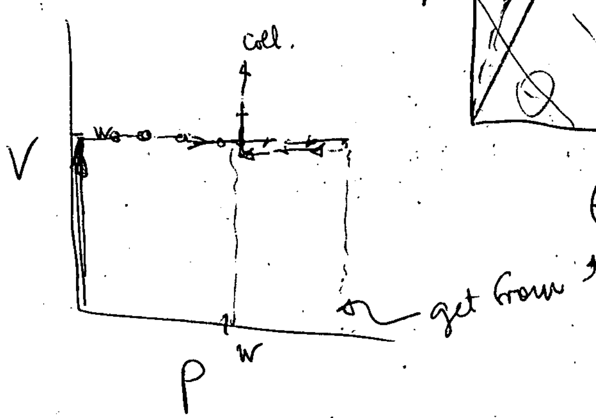
1. Apply increment of loading to the tension jacks. Adjust column load from head of testing machine to provide a stated working stress in the lower portion of the column (It is suggested that this column stress be on the high side, say 18 or 20 ksi, otherwise the column stress in the upper portion of the column will in some cases be very low and therefore unrealistic).
2. Read gages after creep strains are essentially removed.
3. Repeat increments of loading until into the plastic range. When in the plastic range change over to a deflection increment. A suggested procedure is to hold the deflection increment on the specimen for a stated period of time as for example five or ten minutes until the strains and rotations come to rest then take a complete set of readings.

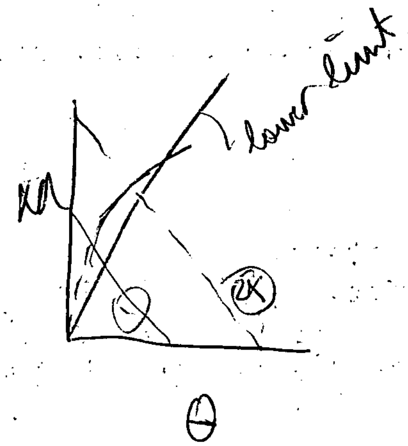
⊗ omit the seat angle

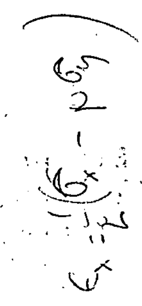




Test scheme







⊗ we decided on this because in phase (α) we would know that this overload on beam will not collapse column. In (β)-phase the column has adequate strength under the working load moment

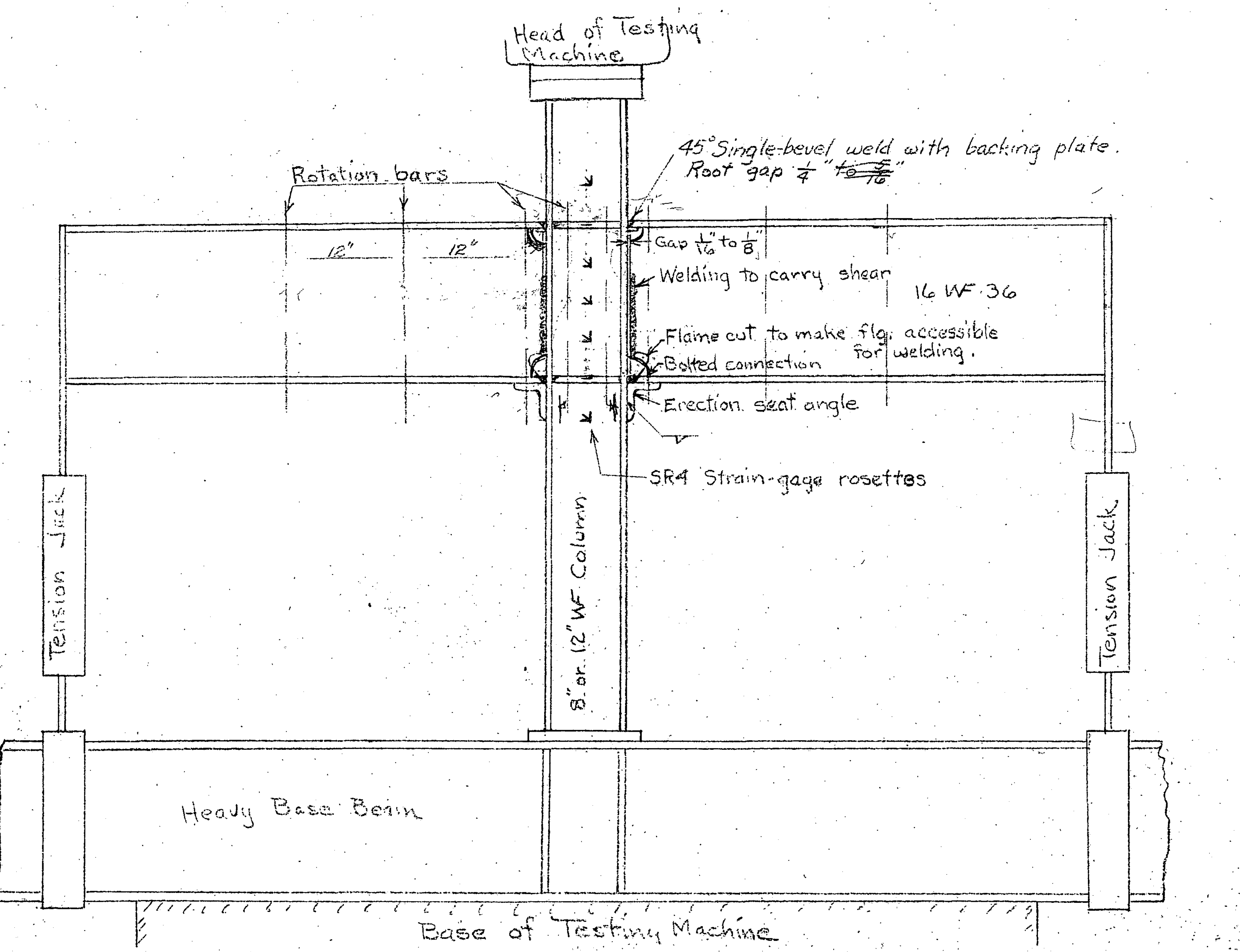


FIG. 1. PROPOSED TEST ARRANGEMENT TWO-WAY BEAM-COLUMN CONNECTION

File 233.7

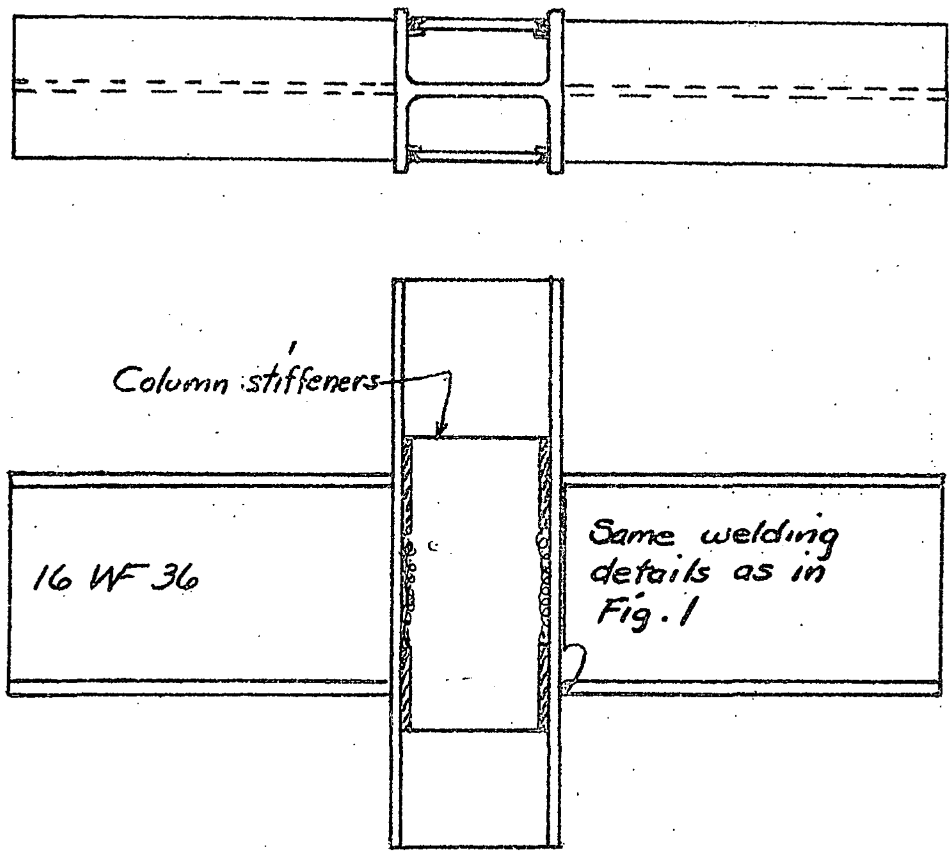


FIG. 1- B. COLUMN STIFFENERS PARALLEL TO WEB OF COLUMN

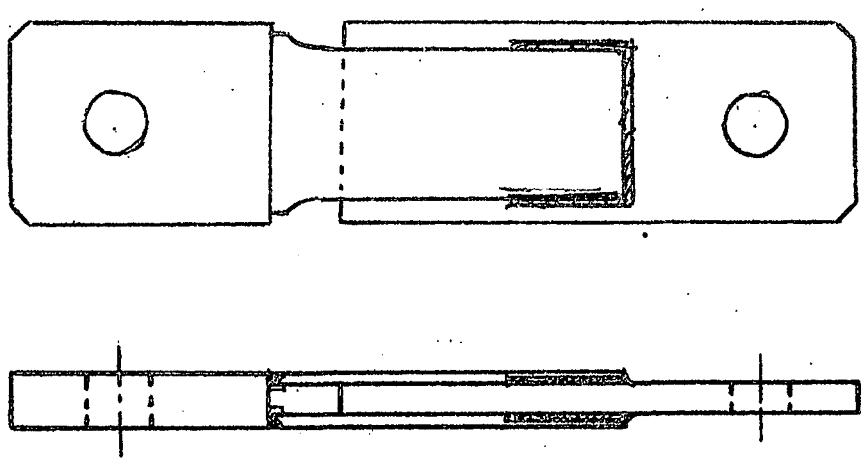


FIG. 2. TOP PLATE TENSIONS SPECIMENS

$120 \times 12 = 1440$

$0\ 33\frac{1}{3} = 500$

Tests

$9 @ 300 = 2700$

$0 @ \frac{1}{2} = 900$

5540